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EXPERIMENTAL CHARACTERIZATION OF SHEAR CONNECTORS FOR STEEL AND CONCRETE COMPOSITE BRIDGE DECKS

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Abstract: The aim of this communication is to show the experimental work developed at University of Minho regarding the characterization of shear connectors' structural behaviour in lightweight concrete and to compare the obtained results with results presented by other authors. The experiments conducted made it possible to determine the load capacity and the deformation capacity of shear studs and Perfobond connectors, and to evaluate its ductility. The advantages or disadvantages related to each type of shear connector and the differences between the technological solutions needed to fabricate and assemble each device are also pointed out.

1. INTRODUCTION

Steel and concrete are materials with different nature and properties. When properly associated, it is possible, in a mechanical point of view, to take the best advantage of each one's properties and of its association, respecting the purpose of a composite structure. Composite action can be obtained by reducing or preventing the relative displacement of concrete and steel elements at their interface. Shear connectors are used to provide this composite action. The behaviour of the steel to concrete connection will influence the global behaviour of the structural element.

A large number of experimental testing has been done in order to explore the structural behaviour of different types of steel connectors. Beside the commonly used headed studs, some investigators showed that the use of Perfobond connectors is adequate when dealing with high strength concrete. Recent experimental work, carried out by [1], [2], [3], [4], [5] and [6] with Perfobond and rib connectors, and studies developed by [7] and [5] with headed studs, made it possible to describe and analyse the steel to concrete connection properties. These studies are primarily focused on normal density concrete (NWC) with normal and high compressive strength. Recent investigation proved that the use of studs is adequate for high strength concrete, [7] and [8]. Headed studs covered with ultra-high strength concrete were also tested, with some improvement on the load capacity and a ductile behaviour. Good results were obtained with lightweight concrete in push-out tests recently performed, [5], [9].

2. SHEAR CONNECTORS

Headed studs are well-known and largely used, in spite of the various types of connection between steel and concrete already studied and commercialised. These connectors are industrially produced and are available in various diameters, usually varying from 6 to 25 mm, and various heights, that can go from 30 to 500 mm.

A description of the load-bearing and deformation behaviour of headed stud shear connectors in standard-strength concrete is given by [10]. According this author, four load-bearing portions are considered: concrete compression strut force behind the weld collar, bending and shearing load-bearing capacity in the lower area of the connector shaft, tensile force in the connector shaft as well as friction forces in the composite interface.

The success of the stud connector has to do with the characteristics of site work: the stud welding is fast, they anchor well in concrete and it is easy to dispose the reinforcement through the slab, between the studs. Other advantages of this device are the facility of massive production, the standard dimensioned head that resists to the slab uplift without extra care and the possibility of being used in steel deck slabs, a constructive system that does not require temporary support and provides extra resistance for positive bending moments.

Some disadvantages can also be pointed out: this shear connection solution demands for equipment that needs high energetic resources to work, the equipment is initially expensive to buy and the welding conditions can be affected by climate conditions at work site. Besides, when embedded in high strength concrete, this connector behaviour is not optimal as there is the possibility of an earlier failure in the composite element. This early failure can occur by fatigue, caused by cyclic loadings that are usual, for example, in bridge decks. Fatigue problems can also occur for service load level.

Moved by the unsatisfactory behaviour of shear studs that result from fatigue problems caused by live loads on composite bridges, the German office *Leonhardt, Andrå and Partners* developed, in the late 1980's, a new type of connector, the Perfobond rib shear connector, [11]. The Perfobond rib shear connector consists on a metallic plate, with a limited number of openings, welded to the steel beam and concreted inside the slab (Figure 1.a). During casting, the plate openings are filled with concrete, forming dowels that provide resistance to horizontal shear and prevent vertical separation between the steel beam and the concrete slab.

The load capacity of a Perfobond connector results from the following parcels, [1]: the tensile strength on the concrete slab, along the Perfobond alignment; the tensile strength of the transversal reinforcement bars; the shear resistance of the confined concrete that lies inside the connector's openings and the bearing of compressed concrete positioned in front of the Perfobond rib. The connector itself usually presents high shear resistance, as the steel plate has sufficient width and length. Thus, the connector shear failure is unusual, contrary to what happens for headed studs and therefore failure usually occurs in concrete. After the concrete dowels failure, the connector still holds considerable shear strength, due to concrete friction at the cracked surfaces that are pressed against each other by the transversal reinforcement, [12].

When compared to headed stud connectors, some advantages can be pointed out for Perfobond connectors: they can be produced in large scale with different shapes and sizes, they can easily be welded without need for special equipment, the welding task can be performed both at site or at an industrial unit, and in terms of load capacity, a significant number of studs can be replaced by a smaller number of Perfobond ribs, as this connector shows a very high load bearing capacity.

In terms of fatigue resistance, Perfobond connectors proved to have better behaviour than headed stud connectors, as the values of slip required to mobilize this connector maximum load capacity are much smaller. If the live load is an important part of the total working load, then slip will occur with every cycle of live load, creating fatigue problems, [12]. Other advantage related to fatigue behaviour is that the connection behaviour until maximum load is essentially elastic, contrary to what happens for headed studs, where an important parcel of plastic slip has already developed when the connection attains the maximum load. In addition, the small longitudinal fillet welds cause smaller residual welding stresses and fatigue problems than the welds of shear studs, [12]. For serviceability loadings, the Perfobond connector usually shows good behaviour, with a much smaller deformation than obtained for stud connectors. This deformation is essentially elastic, [12].

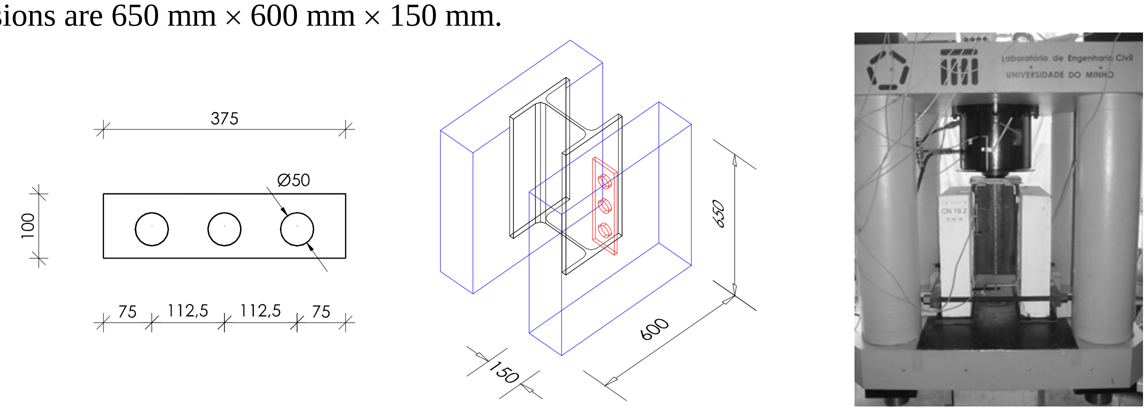
3. THE PUSH-OUT TEST

The push-out specimen consists on a steel beam section held in the vertical position by two identical concrete slabs. The link between the concrete slab and the steel profile is accomplished with steel connectors. Connectors are welded to the steel profile and later embedded on the concrete slab after concreting (Figure 1.b). Chemical or adherence bond between the concrete slab and the steel profile is avoided.

The push-out test was developed to simulate the transmission of forces on a composite beam. The steel profile is subjected to a vertical load, which produces shear load along the interface between the concrete slab and the beam flange on both sides (Figure 1.c). The shear forces applied to the connectors' basis are transmitted to the concrete slab with inclined compression forces, as happens in composite beams.

The push-out test allows for a rigorous analysis on the shear connection behaviour, by assessing the load-slip relation until failure and the failure mechanisms. The choice for the push-put test configuration is adequate as the relations established between forces become simpler than those obtained with a bending test on a composite beam. The shear stresses applied on the connector basis result directly from the forces introduced by the test load cell and it is possible to measure the relative displacement between the steel profile and the concrete slabs during the load application. Results are therefore obtained in a direct way.

The test set up defined for this work follows the EN 1994-1-1 dispositions for shear connection between steel and concrete tests, [13]. For each type of connector, the geometry of the test set-up is similar, with variation on the connector type and disposition. The slab dimensions are 650 mm × 600 mm × 150 mm.



a) The Perfobond connector b) Specimens configuration c) Test setup

Fig. 1: Standard Push-Out Tests

4. HEADED STUDS - EXPERIMENTAL RESULTS AND COMPARISON WITH STANDARD EQUATIONS TO PREDICT THE ULTIMATE LOAD CAPACITY

A good prediction of the characteristic load capacity is important. In order to do so, a check on the expressions currently recommended in the actual codes is presented. The idea is to evaluate the adequacy of using these expressions to evaluate the load bearing capacity of stud shear connectors in high strength lightweight concrete, because these expressions were mainly developed to analyse normal weight concrete shear connection capacity.

Based on the results of experimental tests performed with the standard push-out test, EN 1994-1-1, [13], proposes equations (1) and (2) to calculate the characteristic load capacity value for one single stud. These equations correspond to two possible failure modes: eq. (1) has to do with the shank shear failure and eq. (2) has to do with concrete crushing failure,

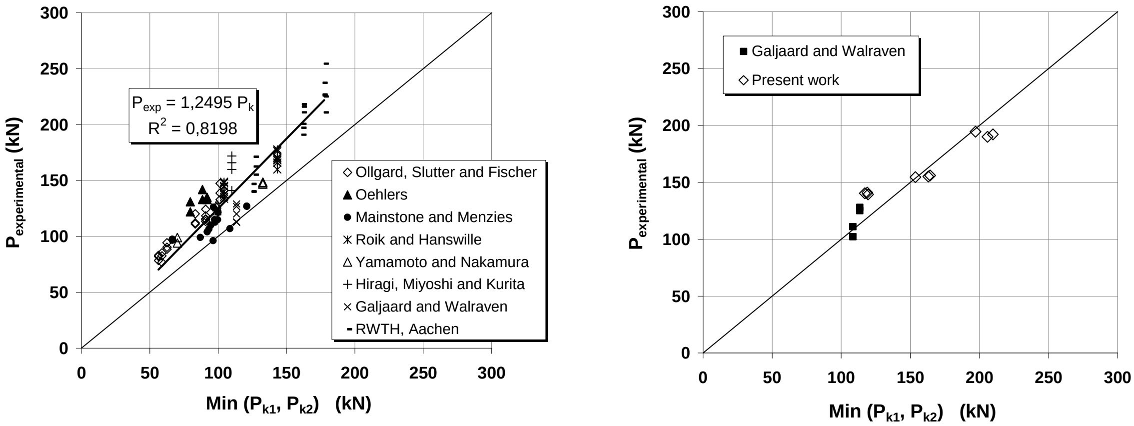
$$P_{Rk} = k \cdot f_u \cdot \frac{\pi d^2}{4} \tag{1}$$

$$P_{Rk} = 0.29 \alpha \, d^2 \sqrt{f_{ck} E_{cm}} \tag{2}$$

where, k = 0.8; d is the stud shank diameter; f_u is the steel tensile ultimate strength for the used studs; and f_{ck} is the characteristic value of concrete compressive strength (in cylinders).

Figure 2.a presents the ultimate loads obtained in push-out tests performed by several authors. These results are compared with the characteristic load obtained by using equations (1) and (2) and considering the characteristics of concrete and steel described by these authors. The tendency is that the characteristic load obtained is smaller than the experimental load and there is a strong linear relation between these parameters. In average, the characteristic loads determined correspond to 80% of the experimental load. This value is in accordance with the dispositions of EN1994-1-1 that calculates the characteristic load from the push-out tests, by considering 90% of the experimental ultimate load.

Figure 2.b presents the same comparison between experimental and characteristic load, but now considering tests performed with lightweight concrete. In fact, the diagrams indicate that the characteristic and the experimental loads are similar, which leads to the following conclusions: (1) when lightweight concrete substitutes normal density concrete there is a tendency to obtain smaller ultimate loads; (2) the equations proposed by EN1994-1-1 should consider a reduction factor for lightweight concrete. If this reduction factor is equal to 0.9, the procedure described to calculate the characteristic loads from experimental tests is reasonable.



a) NWC b) LWC

Fig. 2: Experimental and characteristic load for push-out tests with NWC and LWC

In AASHTO, [14], the shear strength of one headed stud shear connector embedded in a concrete slab is defined by equation (3), where, ϕ is a resistance factor equal to 0.85 and A_{sc} is the cross section of the stud connector shank (in mm^2).

$$P_R = \phi\, 0.5 A_{sc} \sqrt{f_{ck} E_{cm}} \leq \phi\, A_{sc}\, f_u \tag{3}$$

Another expression was proposed by Oehlers and Jonhson, [15], to calculate the shear load capacity for headed studs. Equation (4) was established in an empirical way, but includes the main essential parameters that influence the shear connector load capacity, where, $k = 4.16$ and E_s is the Young's modulus for steel.

$$P_R = k \cdot \frac{\pi d^2}{4} \cdot f_c^{0.35} \cdot \left(\frac{E_{cm}}{E_s}\right)^{0.4} \cdot f_u^{0.65} \tag{4}$$

Table 1 presents the experimental evaluation of load capacity on standard push-out specimens with headed studs, with the equations proposed by [13], [14] and [15].

The steel tensile ultimate strength for studs is determined experimentally and the corresponding results are presented in Table 1. As defined in EN1994-1-1, [13], it is necessary to limit f_u value to 500 MPa to apply equation (1).

Table 1: Characteristic load capacity of headed stud connectors

Specimens	Conc. Ref.	Connectors disposition	Conc. density (kg/m^3)	f_{lcm} (MPa)	E_{lcm} (GPa)	f_u (MPa)	P_k *Exp.* (kN)	P (1) (kN)	P (2) (kN)	P (3) (kN)	P (4) (kN)
CN 19.1	BL17	Single	1899	52.73	24.44				118.8	136.8	125.4
CN 19.2	BL18	Single	1871	52.01	24.06	596	125.4	113.4	117.1	134.8	124.0
CN 19.3	BL19	Single	1914	53.61	24.27				119.4	137.4	125.8
CN 22.1	BL20	Single	1914	55.03	24.51				163.0	180.6	163.9
CN 22.2	BL21	Single	1940	54.76	25.01	559	139.1	152.1	164.3	180.6	164.9
CN 22.3	BL22	Single	1786	53.29	22.48				153.6	176.8	156.6
CN 25.1	BL26	Single	1826	55.61	24.07				209.7	232.4	210.4
CN 25.2	BL27	Single	1819	52.62	24.51	557	171.0	196.3	205.8	232.4	207.9
CN 25.3	BL28	Single	1812	52.69	22.46				197.2	229.9	200.8
CDN 19.1	BL23	Double	1783	54.10	22.80				116.2	133.7	122.9
CDN 19.2	BL24	Double	1854	56.64	27.91	596	107.7	113.4	131.6	143.6	135.6
CDN 19.3	BL25	Double	1816	56.78	25.62				126.3	143.6	131.2

f_{lcm} - mean value of concrete compressive strength, measured in cylinders
E_{lcm} - mean value of modulus of elasticity, measured in cylinders

The results presented in Table 1 and Figure 3 show that all the standard equations give high values for the characteristic load capacity, with exception to diameter 19 mm. It is also significant that this load capacity value tends to diverge from the experimental values as the stud diameter increases.

Equation (2) gives load capacity values that are close to the results obtained with equation (1), showing that for this type of concrete, there is a good balance between connector strength and concrete strength. The tendency verified for equation (1) is repeated, as equation (2) results diverge from the experimental results when the stud diameter gets larger. The results of applying equation (4) follow the same trend as equations (1) and (2). There is no accuracy increase in using this equation.

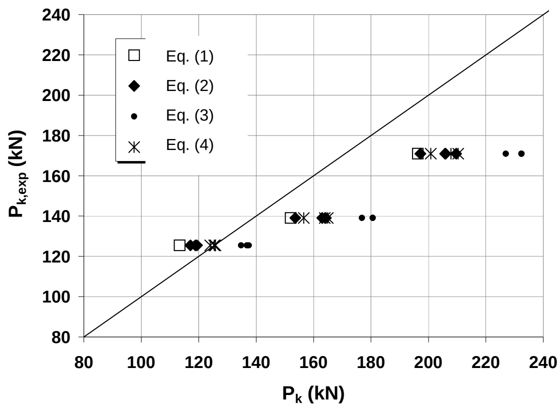


Fig. 3: Comparison between experimental results and standard equations for headed studs load capacity

5. PERFOBOND - EXPERIMENTAL RESULTS AND COMPARISON WITH EQUATIONS TO PREDICT THE ULTIMATE LOAD CAPACITY

Based on a regression analysis made on the results of normal weight concrete specimens with Perfobond connectors of various geometries and reinforcement distribution, Oguejiofor and Hosain, [1], established equation (5) that quantifies the shear connection load capacity.

This expression accounts for the contribution of four essential components. The first parcel considers the concrete slab compressed in front of the rib connector, the second parcel accounts the concrete dowels formed on the connectors' openings, the third parcel evaluates the concrete slab subjected to tensile stresses and the fourth parcel measures the contribution of the transversal reinforcement disposed on the concrete slab,

$$P = B_1\, h\, t\, f_c + \left(B_2\, A_{cd} + B_3\, A_{cc}\right)\sqrt{f_c} + B_4\, A_{tr}\, f_y \tag{5}$$

where, f_c is the concrete compressive strength; f_y is the steel yielding strength; A_{cd} is the concrete shear area (inside the connectors' openings, $A_{cd} = n \cdot \pi \cdot d^2 / 4$)); A_{cc} is the concrete shear area (outside the connectors' openings); t is the Perfobond rib width; h is the Perfobond rib high; d is the diameter of the Perfobond rib openings; n is the number of Perfobond rib openings and A_{tr} is the area of transversal reinforcement.

Initially, the adjusted equation presented the following factors: $B_1 = 0$, $B_2 = 3.655$, $B_3 = 0.59$ and $B_4 = 1.233$. After performing more experimental tests and using numerical models, a new equation was proposed by the same authors to calculate the connection load capacity, with the following factors: $B_1 = 4.47$, $B_2 = 4.20$, $B_3 = 0.01$ and $B_4 = 0.91$. This new equation is based on the results of a regression analysis performed on a large number of results obtained from a numerical model calibrated from experimental results and is confirmed with more experimental tests. The equation maintains the contribution of the concrete dowels, the contribution of the transversal reinforcement, the contribution of the slab tensile strength and adds the effect of localized compression in front of the Perfobond rib (1^{st} parcel).

Later, [16] tested a large number of push-out specimens with a new type of shear connector, the CRESTBOND connector, that is an indented connector formed by a rib that is similar to the Perfobond rib, but with open apertures. The experimental results obtained with the Crestbond connector showed that:

- to consider the ratio between the transversal reinforcement and the concrete slab transversal area gives better results than to consider only the transversal reinforcement area;

- the connector eccentricity in relation to the concrete slab height should be considered in the analytical model; this is done by multiplying the force developed in front of the rib connector by the ratio between the connector height and the concrete slab height (h_{sc}/t_c).

Considering the similarities observed between Perfobond and Crestbond specimens in terms of failure mechanisms, the authors made a new study in order to define an equation that could properly quantify the connection load capacity for both connection types. The new adjusted equation has the form presented in equation (6).

$$P = B_1 \frac{h_{sc}}{t_c} h_{sc}\, t_{sc}\, f_c + B_2\, n\, D^2 \sqrt{f_c} + B_3\, A_{cc} \sqrt{f_c} + B_4 \left(\frac{A_{tr}}{A_{cc}} \right) \tag{6}$$

Then, they performed a new multiple regression analysis on the results presented by [1] with the equation proposed. The multiple regression analysis performed gave the following coefficients: B_1 = 4.044, B_2 = 2.369, B_3 = 0.157 and $B_4 = 31.85\times10^6$. The adjusted correlation coefficient is equal to 0.995. This equation applied on the results of [1] gives results that are closer to the experimental ones than the results obtained with equation (5) that was proposed by the same authors.

Table 2 presents the results obtained by the present authors for two series of tests with Perfobond connector and lightweight concrete: Series CP X.1 and Series CP X.2, where A_s is the transversal reinforcement area passing through the connectors' openings and A_d is the transversal welded wire mesh disposed on the slab's upper face.

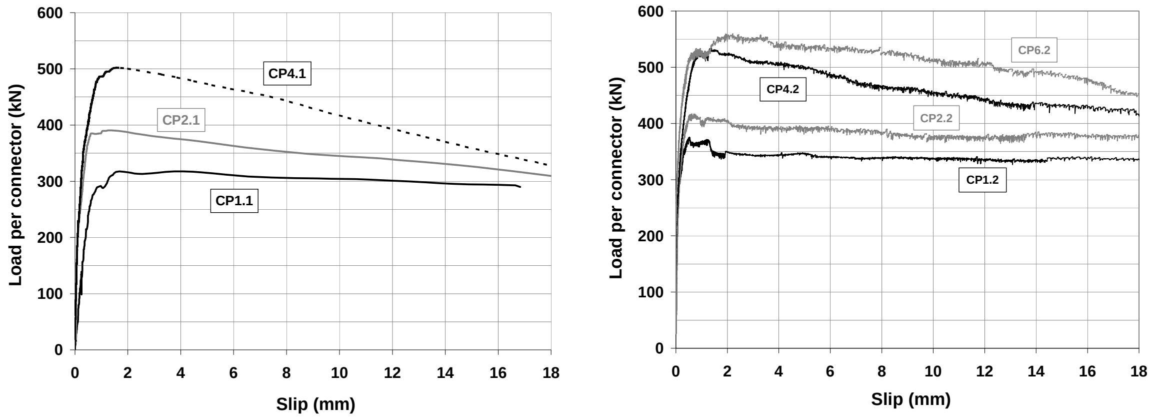
Table 2: Experimental results for Perfobond connector

Specimen Ref.	P_{max} (kN)	P_k (kN)	s (P_{max}) (mm)	$s_{elast,90\%}$ (mm)	$s_{plast,90\%}$ (mm)	$s_{total,90\%}$ (mm)	s_{ki} (mm)	A_s (cm^2)	A_d (cm^2)
CP1.1	317.7	285.9	1.676	0.792	16.041	16.833	14.437	0.000	0.000
CP2.1	390.6	351.5	1.390	0.426	7.833	8.259	7.050	0.785	0.000
CP3.1	237.7	213.9	2.197	0.907	8.399	9.306	7.559	2.356	0.000
CP4.1	502.1	451.9	1.575	0.684	6.706	7.391	6.036	2.356	0.000
CP1.2	375.1	337.6	0.532	0.264	18.133	18.396	16.319	0.000	1.178
CP2.2	416.8	375.1	0.615	0.315	23.619	23.934	21.257	0.785	1.178
CP4.2	533.6	480.3	1.364	0.543	6.126	6.669	5.513	2.356	1.178
CP5.2	311.0	277.0	7.874	3.131	17.035	20.166	15.332	0.000	1.178
CP1.2	375.1	337.6	0.532	0.264	18.133	18.396	16.319	0.000	1.178

Figure 4 presents the load-slip curves for specimens from series CP X.1 and CP X.2, for specimens where only the transversal reinforcement is varied. Although the first series were not tested with deformation control, the general behaviour is similar for all specimens. Some principal aspects, common to both series can be pointed out from the analysis of Table 2 and Figure 4:

- the initial phase of loading is very stiff for all the tested specimens;
- the load-slip behaviour can be considered as elastic almost until the maximum load;
- maximum load is attained for very small deformation values, with exception to specimen CP5.2, in which the connector openings are suppressed;
- after the maximum load value is attained, the load decreases very slowly;
- all the specimens maintain high load capacity for large deformation values, well beyond the slip measured for maximum load;
- after the maximum load value is attained, the load decrease is more pronounced for specimens with higher transversal reinforcement area;

- elastic slip is considered for 90% of the maximum load value and is similar for both series, with exception to specimen CP5.2;
- the specimens with no transversal reinforcement tend to lose load capacity slower than other specimens, although the maximum load value is smaller.



a) Series CP X.1 b) Series CP X.2

Fig. 4: Comparison of load-slip curves for Series CP X.1 and CP X.2

5.1 Influence of concrete bearing in front of the connector edge

In order to evaluate the influence of concrete bearing in front of the connector edge, experimental results obtained within this work are compared with the results obtained by Ferreira, [2]. The tested specimens are similar for both experimental studies, with small differences regarding the connector height and the concrete slab dimensions and a more important difference that has to do with concrete compressive strength (Table 2). For comparison purposes, two types of specimens are tested: one with a plain slab and other with a longitudinal opening that goes from the bottom of the slab until the Perfobond rib basis.

The values presented in Table 2 show that, for NWC, the results of quantifying the influence of localized concrete compression in front of the connector edge with equation (5) are slightly lower than results obtained experimentally. The opposite happens for LWC: the experimental load result corresponds to approximately 80% of the load predicted with the first parcel of equation (6), which means that this equation should be modified in order to better account the effect of localized compression when LWC is used.

Table 2: Experimental results for Perfobond connector – difference between CP4.1 and CP3.1

							Concrete slab		Perfobond rib		
Speci-men	Specimen Type	Con crete	$f_{lcm,Slab1}$	$f_{lcm,Slab2}$	P_{max}	ΔP	L	H	H	t	1st parcel from (6)
			(MPa)	(MPa)	(kN)	(kN)	(mm)	(mm)	(mm)	(mm)	(kN)
CP4.1	Plain slab	LWC	65.67	62.81	502.1	264.4	650	150	100	13	317.5
CP3.1	With opening**	LWC	60.13	60.80	237.7		650	150	100	13	337.4
PB06 (*)	Plain slab	NWC	9.65		278.0	62.5	720	100	80	12.7	44.1
PB07 (*)	Plain slab	NWC	10.00		274.4		720	100	80	12.7	45.7
PB05 (*)	With opening**	NWC	12.80		220.1		720	100	80	12.7	58.5
PB08 (*)	With opening**	NWC	11.73		207.4		720	100	80	12.7	53.6

* – specimens tested by Ferreira [2]

** – the slab opening is in front of the rib connector

5.2 Influence of concrete dowels passing inside the connectors' openings

The difference between specimens CP5.2 and CP1.2 are the openings on the Perfobond rib. The Perfobond connector used has three circular openings with 50 mm diameter. The difference between these two specimens is the contribution of the concrete dowels in terms of load bearing capacity and deformation. Table 3 expresses the results obtained for these specimens.

Figure 4 plots the experimental values presented in Table 3 based on calculations made with the second parcels of equations (5) and (6) and the results obtained by [1] that obtained comparable experimental results considering Perfobond connectors with very similar geometry, but now using NWC. As done for specimens CP5.2 and CP1.2, two similar specimens, one with Perfobond rib with three openings and the other with steel rib without openings are compared and the respective maximum loads are subtracted.

Table 3: Experimental results for Perfobond connector – influence of concrete dowels

Specimen Ref.	$A_{c,openings}$ (cm^2)	P_{max} (kN)	CP1.2 – CP5.2 (kN)	s_{Pmax} (mm)	$s_{total,90\%}$ (mm)
CP1.2	58.905	375.1	64.2	0.532	18.396
CP5.2	0.0	311.0		7.874	20.166

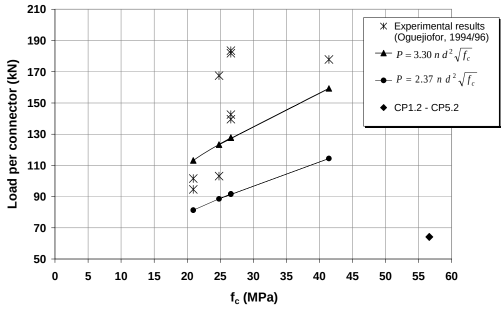


Fig. 4: Evaluation on the concrete dowels load capacity, for NWC and LWC

This comparison shows that the load capacity of the specimen with openings is higher. The difference in load capacity between the two specimens is equal to 64.2 kN. The average shear strength experimentally determined for LWC is equal to 3.71 MPa. For specimen CP1.2, a total concrete shear area of 117.8 cm^2 corresponds to the connectors' holes, considering that concrete shear failure occurs in both sides of the connector rib. A prediction on the concrete dowels load capacity contribution is obtained by multiplying these two values. The result is equal to 43.7 kN. The predicted value corresponds to 68% of the experimental result, which probably means that there is a higher concrete confinement on the push-out specimen, provided by the layer of welded wire mesh and the transversal reinforcement. However, there is a good proximity between these two results.

There is a great variability associated with the results obtained by [1]. To define equation (5), they performed more experimental tests, varying also the number of the connector openings and studied a numerical model calibrated with the experimental results.

The result obtained with specimens CP1.2 and CP5.2 is much smaller than the results experimentally obtained by [1] and the results obtained with equation (5). One possible reason is that LWC shear strength is lower than NWC shear strength, which should be considered.

5.3 Influence of the transversal reinforcement passing inside the connectors' openings

Table 6 displays the connection maximum applied load, in relation to the transversal reinforcement area passing through the connectors' openings, for Series CP X.1 and Series CP X.2. The linear relation that better fits the obtained results is plotted for each series. Both series show that there is a strong linear relation between the connection load capacity and the transversal reinforcement area. The reinforcement contribution on the connection load capacity can be related to the reinforcement yielding strength, as presented in Table 6, where f_y is the steel yielding tensile strength and A_s is the transversal reinforcement (m^2).

Table 6: Load capacity contribution obtained with reinforcement bars passing through the connectors' openings

Type of specimen	Concrete type	Reinforcement diameter - *d* (mm)	POST tests f_y (MPa)	POST tests *P* (kN)
Series CP X.1	LWC	10	576	$P(A_s) = 1.38 \cdot f_y \cdot A_s$
Series CP X.2	LWC	10	576	$P(A_s) = 1.01 \cdot f_y \cdot A_s$
	LWC	12	523	$P(A_s) = 1.12 \cdot f_y \cdot A_s$
C2 , C2-R, [1]	NDC	10	478	$P(A_s) = 1.71 \cdot f_y \cdot A_s$
C3 , C3-R, [1]	NDC	10	478	$P(A_s) = 1.30 \cdot f_y \cdot A_s$

The same approach is followed for some of the results obtained by [1], regarding four specimens with the same concrete compressive strength and geometric disposition, where the area of transversal reinforcement is varied (Table 6). The Perfobond connector used by these authors is very similar to the Perfobond connector used within this work. The results obtained are in agreement with the ones here obtained, which means that the contribution of transversal reinforcement on the connection load capacity is not altered by substituting NWC with LWC.

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